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Water-Surface Elevation Frequency Estimates for the Louisiana Coast

by S. Rao Vemulakonda,[1] Norman W. Scheffner,[1] David J. Mark,[1] and Mitchell E. Brown[2]

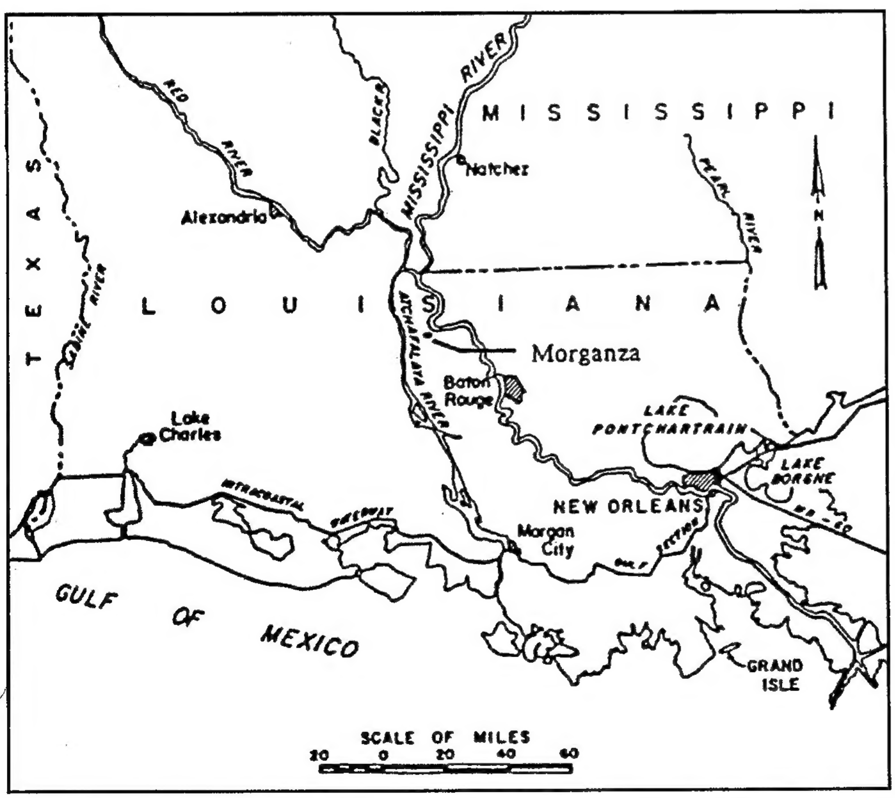

Figure 1. Study area (after U.S. Army Engineer District, New Orleans 1980)

Introduction

The coast of Louisiana (Figure 1) generally consists of low-lying land with extensive marshes and interconnected bayou systems. Historically, this area has been prone to extensive flooding and property damage because of both tropical (hurricanes) and extratropical storms. Over the years, a series of levees and control structures have been constructed to reduce the impact of storms. Recently the U.S. Army Engineer District, New Orleans, embarked on two related studies: (a) the Lake Pontchartrain Study to investigate storm impacts along the open coast of Louisiana, and (b) the Morganza Study on the present and future uses of the area from Morganza to the Gulf of Mexico. Both studies require estimates of stage-frequency relationships at different locations.

The U.S. Army Engineer Waterways Experiment Station (WES) Coastal and Hydraulics Laboratory applied the two-dimensional (2-D) finite-element, long-wave hydrodynamic model ADCIRC-2DDI (Westerink et al. 1992) to the study area. The model was modified to account for wetting and drying of land because of flooding. ADCIRC was calibrated and verified using field data measured at several locations during Hurricanes Betsy (1965) and Andrew (1992). The model-generated surge levels accurately matched the measured data. Subsequently, the model was applied to several historic storms, and computed maximum surges and time series of surges were saved at

[1] Research Hydraulic Engineer, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.
[2] GIS Analyst, Mevatec Corporation, Vicksburg, MS.

gauge locations of interest. A stage-frequency analysis was performed for the gauge locations using the Empirical Simulation Technique (EST) (Scheffner, Borgman, and Mark 1996). Conceptually, this statistical approach considers storm characteristics as the input to the system and peak surges as the response.

ADCIRC Numerical Model

ADCIRC is a 2-D, depth-averaged model that uses the Generalized Wave-Continuity Equation approach to solve the equations of momentum and continuity. It employs numerical discretizations using the finite element method in space and finite difference method in time. It allows for extreme grid flexibility that permits simultaneous regional/local modeling and is both highly accurate and efficient.

The wet/dry algorithm operates on a fixed grid, with whole individual elements being either wet or dry. Conceptually, a dry element has barriers along all of its sides, and the barriers are removed as the element is flooded. Nodes may be classified as dry, interface, or wet. Interface nodes are connected to both dry and wet elements and are similar to the standard land/water boundary nodes. A no-slip boundary condition is implemented at these nodes. A node dries when the depth falls below a user-specified minimum depth (e.g., 0.91 m (0.3 ft)). However, in order to minimize numerical noise, a node must remain wet for a minimum number of time-steps (e.g., 10 to 20) before it can dry. A similar constraint is imposed for wetting of dry nodes. An interface node is reclassified as a wet node if the water-level gradient and the vector sum of the water-level gradient and the wind stress both favor water transport towards all the dry nodes connected to that interface node. Furthermore, a sufficient quantity of water must reside in the wet elements composing the flood/dry interface. When the interface node is reclassified as wet, all dry nodes connected to that node become interface nodes.

An existing ADCIRC grid of the Gulf of Mexico was modified for the study by adding finer resolution in the study area. The enhanced numerical grid had 25,732 nodes and 50,215 elements. Boundary conditions were clearly posed and away from the project area of interest. In this case, there are two open-water boundaries, one across the Strait of Florida and the other across the Yucatan Channel. Grid refinement extends from the Gulf of Mexico shoreline south of New Orleans to Interstate Highway 10 approximately 96.56 km (60 miles) north. The refined area includes Lake Pontchartrain and Lake Borgne, and Atchafalaya Bay to the west. Minimum nodal spacings are on the order of 61 m (200 ft). The smallest elements are located near Chef Menteur Pass, which connects Lake Borgne with Lake Pontchartrain.

In addition to wind forcing, tidal forcing was used for calibrating and verifying ADCIRC to historic hurricanes. Five tidal constituents (M_2, S_2, K_1, O_1, and P_1) were modeled. For each constituent, amplitude and phase (nodal factor and equilibrium argument) were furnished at the two open boundaries using results of a larger hydrodynamic model that included in its domain the western North Atlantic Ocean, the Gulf of Mexico, and the Caribbean Sea. In addition, ADCIRC used tidal potential forcing at all grid nodes.

For tropical storms, the National Hurricane Center's hurricane database was used to determine the track and characteristic parameters (eye location, maximum wind speed, forward speed, minimum pressure, etc.) of a historical hurricane as a function of time. This information was used as input to the Planetary Boundary Layer model that computed wind and atmospheric pressure fields on an hourly basis. This information was archived to data files and used as input to ADCIRC. For extratropical storms, wind information (velocity components) was obtained from the U.S. Navy Fleet Numerical Meteorological and Oceanographic Center's database every 6 hr at 2.5-deg latitude-longitude spacing and used as input to ADCIRC, which interpolated the winds to grid nodes as needed in time.

EST Procedure

The EST is a statistical procedure for modeling frequency-of-occurrence relationships for nondeterministic multiparameter systems. In this case, it was used to develop the frequency relationship for the maximum storm surge elevations as a function of storm characteristic parameters. Tropical and extratropical storms were considered separately because of their different characteristics. In applying the EST, multiple statistical simulations are conducted, each having a duration of N years (e.g., $N = 200$), at each of the selected stations where frequency relationships were desired. Because the procedure is repeated multiple times, an average stage-frequency relationship is generated together with the standard deviation that provides a measure of the uncertainty contained in the relationship. This, coupled with the fact that the EST preserves the statistical relationships inherent in the historical data without relying on assumed parametric relationships or assumed parameter independence, makes the method superior to traditional techniques such as the joint-probability method.

ADCIRC Calibration and Verification

ADCIRC was calibrated using water levels for Hurricane Betsy and verified with water levels for Hurricane Andrew because these storms were severe and because extensive field measurements were collected after their occurrence. No separate calibration was performed for extratropical storms. Typically for tropical storms, simulations were started approximately 2 days before the storm made landfall, and the simulation was conducted for 4 days. For extratropical storms, the simulations were performed for

8 days because of the longer duration of these storms. Generally, a time-step of 5 sec was used. In the wet/dry routine, the minimum water depth was set at 0.3 ft, and wet and dry nodes were kept in their respective states for a minimum of 12 time-steps.

Because the primary interest of the study was in the maximum surges at different gauge locations, time series information on surges was saved at 10-min intervals to resolve the maximum computed surges. Also, surges over the entire grid were saved at the same interval, and the solution was examined using visualization software. Maximum computed surges (maximum for the duration of the storm) were determined at all grid nodes, and contours of maximum surge were plotted and examined. Table 1 shows a comparison of computed and observed peak surges at select stations distributed around the project area for calibration and verification. The agreement is excellent, considering the uncertainty in datum levels in this region.

Results

Following calibration and verification, 19 hurricanes and 11 extratropical storms were simulated using ADCIRC, and the results were archived for use in the EST. At each location of interest, 100 statistical simulations of a 200-year sequence of storms were performed. The results were used to develop separate frequency-of-occurrence relationships for tropical and extratropical storms. The two relationships were then combined into one relationship applicable to both storms. As an example, the results obtained for the Rigolets are shown in Figure 2. Even though the procedure and the tools described here were used previously, this study posed special challenges in terms of the complexity of the region, bathymetry and topography, and wetting/drying. These were successfully overcome, and all of the study objectives were met. This was the first WES study to use ADCIRC's new wetting/drying feature that worked very well. Additional details on this study may be found in Vemulakonda et al. (1997).

Table 1. Comparison of Maximum Surges (ft) at Select Stations for Hurricanes Betsy and Andrew				
	Betsy		Andrew	
Name	**Obs.**	**Comp.**	**Obs.**	**Comp.**
Pascagoula	5.4	5.0		
Biloxi	7.6	7.7		
Rigolets at US 90	7.0	7.5	3.7	3.8
Causeway/North	5.1	6.0		
Causeway/Midlake	3.5	4.6	4.4	4.2
Blind River	4.2	5.1		
Ruddock	10.2	9.2		
Westend	5.6	5.1	4.1	3.3
Seabrook	4.4	3.5		
Chef Menteur at US 90	9.1	8.3	4.2	4.5
Shell Beach	9.3	7.4	5.3	5.2
Breton Sound			5.3	5.4
Phoenix	8.3	8.7		
Catfish Lake			5.9	5.6
Port Eads	3.0	3.4		
Grand Isle	4.5	4.7	3.9	4.2
Leeville	5.5	6.2	5.8	5.1
Bayou Petite Caillou			8.3*E	7.1
Eugene Island			3.8	3.4
Note: Obs. = Observed; Comp. = Computed; *E = Estimated.				

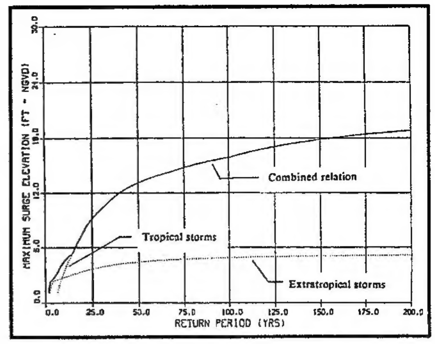

Figure 2. Stage-frequency estimates for the Rigolets at US 90

Acknowledgments

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Post-Jetty Inlet Shoal Evolution, Ponce de Leon, Florida

by Donald K. Stauble[1]

Introduction

Waves, tidal currents, and longshore drift influence the morphologic evolution of ebb and flood shoals, the throat section, and adjacent shorelines at inlets. Complex interactions of these physical forces, with sediment transport processes, control erosion and deposition patterns of shoal features, channel position, and adjacent shoreline response. Variations in these processes can occur between natural and structured conditions within the distinct morphodynamic environments of the inlet. Studying the variations in deposition patterns both in the natural state and after jetty construction is needed to improve understanding of inlet processes and inlet engineering modification.

Ponce de Leon Inlet was evaluated because of its large long-term data set of morphologic evolution for both pre- and post-jetty conditions. The inlet exhibited an ebb shoal that had a symmetric crescent shape in its natural pre-jetty configuration. After jetty construction, the ebb shoal formed an asymmetrical downdrift offset starting at the north jetty, with the center of mass located off the southern beach. Based on Hayes (1979) classification scheme, the mean tidal range and mean wave height place the inlet in a mixed energy zone on the border between wave- and tide-dominated coasts. The correlation between minimum cross-sectional area and tidal prism work of Jarrett (1976) indicates the inlet has a similar relationship to other inlets such as Barnegate Inlet, New Jersey. Tidal prism and longshore drift characteristics of Bruun and Gerritsen (1960) characterize the inlet as a bar-bypasser.

The inlet shoals, adjacent shoreline, and channel configurations have been accurately documented from widely diverse data sets using GIS. The ability to register all data into a common datum and coordinate system has provided a method to assess both the natural and engineered morphodynamics. Bathymetric data were digitized from National Oceanic and Atmospheric Administration and Corps District bathymetric surveys and Scanning Hydrographic Operational Airborne Lidar Survey (SHOALS) surveys (Irish, Parson, and Lillycrop 1995). They were converted into a common vertical datum (NGVD29) and horizontal datum (Florida East State Plane) using Plus3 Software TERRAMODEL. Shoreline position data were obtained from the State of Florida, Department of Environmental Protection. Aerial photography from various sources was scanned and rectified to a common scale using INTERGRAPH Microstation Base Imager.

Inlet History

Ponce de Leon Inlet is located along the central Florida Atlantic coast about 80 km north of Cape Canaveral (Figure 1). This is the only inlet to drain the Halifax River Lagoon to the north and is the principal inlet for the Indian River/ Mosquito Lagoon complex to the south. The first record of the inlet was a river entrance found by Ponce de Leon in 1513. The inlet was given its present name in 1926. In 1932/33, the Intracoastal Waterway was rerouted to the west by dredging a bypass channel because of problems navigating through the highly changeable inlet flood shoal. The inlet became non-navigable over the years, and a pre-south-jetty-construction survey was conducted in 1967. The south jetty was constructed in 1968/69, and the north jetty was completed in 1971 with a 549-m-long weir section.

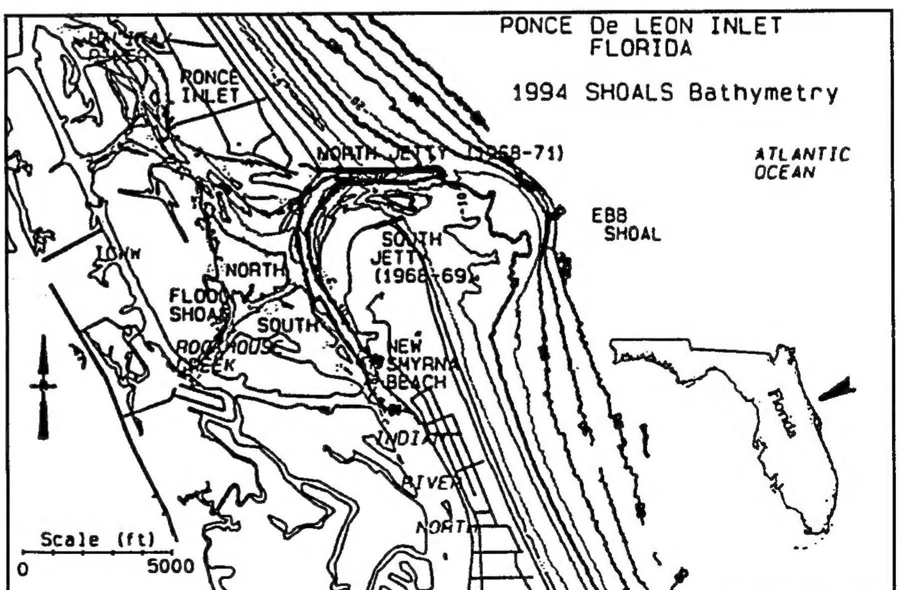

Figure 1. Location map of Ponce de Leon Inlet, Florida, showing shoal and structure features

[1] Research Physical Scientist, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Post-Jetty Morphology

The first post-jetty bathymetry available was an incomplete survey of the flood shoal, throat, and northern shelf area in 1978. Sand accumulation on the south side of the south jetty produced a downdrift offset of the shoreline. A large spit off the north beach and a spit from a previous south swash platform formed an elongated throat channel. The main navigation channel was back to the 1925 northeast-southwest orientation, but was forced to the east when it intersected the north jetty. In the 7 years since jetty construction, the ebb shoal increased in volume and became asymmetrical. With the focusing of the flow by the jetties, the ebb-shoal crest moved seaward about 550 m, and to the south of the north jetty and the navigation channel.

Bathymetry from 1981 and 1986 covering the flood shoal and throat showed a reorientation of the throat channel, most likely because of the weir closure in 1983/84. The navigation channel became more narrow and moved more to the north and west. The point of deflection of the channel along the north jetty moved landward along the north jetty. Ebb-shoal growth continued seaward and to the south, with an increase in volume.

The present configuration of the inlet was ascertained by a 1994 SHOALS survey. The inlet throat has continued to migrate to the north and west, intersecting the north jetty closer to shore, forming a large scour hole in that vicinity. A shoal has also developed on the south side of the throat between the two jetties. The ebb shoal has not advanced seaward, but continues to grow to the south while increasing in volume.

Shoal Evolution

Just prior to jetty construction in 1967, growth of a northern barrier spit and formation of a southern swash platform caused the main throat channel to reorient further to the east. The ebb shoal was still symmetrical but at its most southern position. At this time, the northern flood channel was the dominant channel, with the south channel almost shoaled in. After jetty construction, the southern shoreline quickly filled to the southern jetty. By 1978, the throat channel had reoriented to its 1925 northeast orientation within the throat, but was deflected to the east by the north jetty. The ebb shoal rapidly became asymmetrical to the south and moved seaward as the flow was funneled along the north jetty. The north spit continued to grow into the lagoon as sand moved over the north jetty inner weir section, and the large swash platform welded to the southern shore. This sedimentation moved the dominant north and shallow south flood channels further to the west. With the growth of the south beach out to the limits of the south jetty, the system began to perform as a single jetty system (Kieslich 1981), forcing the channel closer to the longer north jetty.

By 1981, the northern edge of the ebb shoal had migrated south, even with the north jetty. The weir was closed in the mid-1980s, and by 1986, the southern flood channel returned to the dominant channel and began to migrate eastward into the inlet, eroding the southern spit. With the closing of the weir, the throat channel continued to migrate to the north and west, impacting the north jetty at the old weir position in 1990. By 1994, a shoal formed on the south side of the throat, as the throat channel had migrated against the north jetty. The ebb shoal continues to migrate to the south, with the center of mass some 760 m south of the inlet.

With jetty construction, the inlet cross-sectional area decreased as the channel migrated to the north and west against the north jetty. Ebb-shoal volumes were measured based on a common datum of -12 m within a common area shown in the ebb-shoal box of Figure 2. This is a conservative measure of ebb-shoal volume as the shoal moved seaward and the center of mass migrated south. Limits in post-jetty survey coverage on the ebb shoal to the south because of breaking waves restrict total volume calculation but give an estimate of relative ebb-shoal volume change. Flood-shoal volume was calculated in the box identified in Figure 2 using a -6 m datum, based on cross-section profiles. The volume increased as the channel migrated through the box to a maximum in 1967 as the north and south beach spits accreted. A decreasing trend was measured to a low in 1986 when the channels moved west. Present volume has increased again with the formation of the triangular north and south flood shoal and channel migration east, back toward the inlet.

Conclusions

Since completion of the jetty system, the ebb shoal has undergone major evolutions, with seaward growth, increased volume, and a more asymmetrical shape. At the present time, the ebb shoal is south of the north jetty, with the center of mass adjacent to the south beach. The channel, now restricted by jetties, is attempting to return to pre-jetty orientation within the confines of the structures that force the channel to the dominant north jetty. The flood-shoal channel has been dynamic, with large changes in flood-channel position and dominance as the inlet has evolved.

Understanding inlet geomorphic change and channel shoaling has become an important part of inlet engineering. Erosion of adjacent shoreline, sources of beach restoration material, and improvements to navigation are all related to an understanding of shoal evolution.

Acknowledgments

This research is part of the Coastal Inlets Research Program, focusing on long-term inlet morphology and channel evolution. Additional details are provided in Stauble (1998). The ultimate goal of the research is to develop mitigative or engineering designs that will provide a stable navigation channel with little adverse impact on adjacent shorelines.

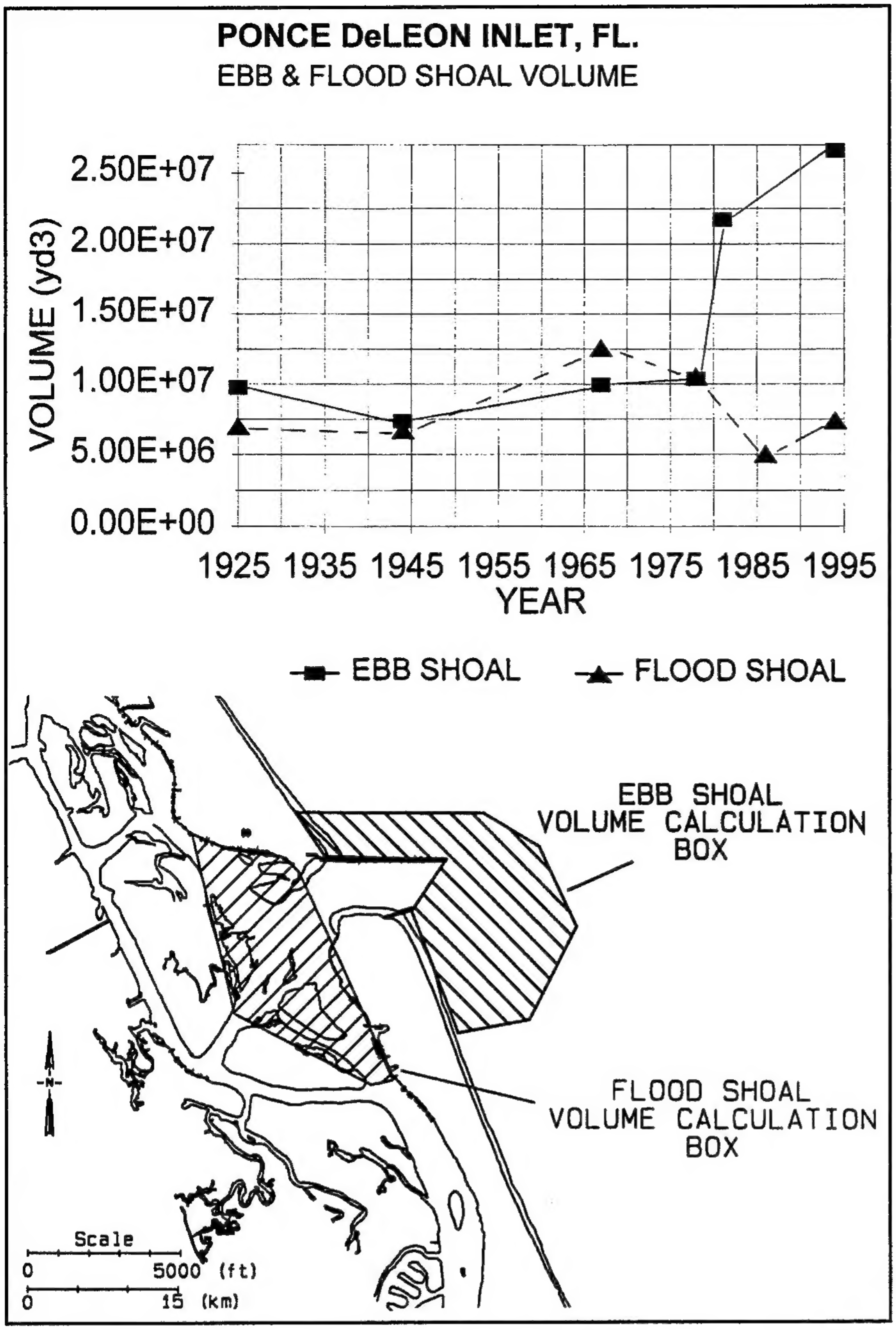

Figure 2. Ponce de Leon Inlet ebb- and flood-shoal volume change

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Numerical Simulation of Beach-Profile Change Accounting for Hard-Bottom Features

by Nicholas C. Kraus[1] and Magnus Larson[2]

Introduction

A hard bottom is a nonerodible sea-bottom feature that may be located anywhere on the wet or dry beach. Modern design of beach and dune complexes normally involves application of a numerical model of beach erosion to evaluate the level of protection afforded by design alternatives (Larson and Kraus 1991). Hard bottom alters straightforward calculation of a movable bed. It can restrict sediment movement because the area it occupies does not contribute to the sediment budget. Calculations performed as if the hard bottom were not there could indicate erosion of beach faces and dunes that cannot erode. Designers need to know if cross-shore movement of sediment will cover hard bottom. If hard bottom (e.g., coral) is predicted to be covered by material from a beach nourishment project or by planned placement of dredged material, mitigation measures might be planned concurrently or an alternative design considered.

The U.S. Army Engineer District, Jacksonville, requested enhancement of the Storm BEACH (SBEACH) numerical simulation model (Larson and Kraus 1989) to account for the presence of hard bottoms in computing dune and beach erosion. This capability was needed by the District to deal with various forms and types of hard bottom present in its beach-fill and dredged-material placement projects along Brevard, Martin, St. Lucie, and Indian River counties, Florida.

The technical background of the hard-bottom enhancement to SBEACH is described in a recent report (Larson and Kraus 1989). An overview of selected aspects of the implementation and verification of the model is presented by Kraus and Larson (1998). The enhanced model allows burial and uncovering of hard bottom, depending upon the wave conditions and sediment supply.

Physical Situation

SBEACH computes dune erosion and beach-profile change as produced solely by cross-shore sediment transport during high waves and elevated water levels of storms. The modeled beach should exhibit uniformity alongshore for most reliable results. Figure 1 shows exposed hard bottom on the beach face and higher on the berm (left side of picture). The hard bottom on the upper beach would act as a seawall, preventing sediment from being removed from the landward side of the beach. Hard bottom prevents lowering of the beach level, but it does not alter the sediment-transport rate and profile evolution until it becomes exposed. Hard bottom therefore functions comparably in controlling beach-profile evolution as does a seawall that controls shoreline response produced by gradients in longshore sediment transport.

Concept and Numerical Implementation

In the hard-bottom calculation algorithm, the potential net cross-shore transport rate is first calculated at all grid points across the profile without considering the presence of a hard bottom. If hard bottom is present at a particular location or will become exposed during the calculation time-step, constraints are placed on the transport rate so that the profile elevation remains fixed along profile

Figure 1. Exposed hard bottom on the beach face and upper berm

[1] Research Physical Scientist, U.S. Army Engineer Waterways Experiment Station, Coastal and Hydraulics Laboratory, Vicksburg, MS.
[2] Associate Professor, Department of Water Resources Engineering, Lund Institute of Technology, University of Lund, Sweden.

segments where the hard bottom is exposed. (It is assumed that the hard bottom is nonerodible; a modification would allow slower erosion, as for a clay sublayer.) By employing the sand-volume conservation equation, calculated depth changes indicate where hard bottom may restrict the transport and profile change. The equation is conveniently solved on a staggered grid where the elevations are defined in the middle of a calculation cell and the transport rate at the boundaries.

The elevation of the hard bottom must be known at all grid points across shore. The hard bottom can only influence points downdrift (in the direction of transport at a given time-step) of the location where it is exposed. An algorithm for correcting the transport rate must not only identify points where hard bottom constrains the transport but, because the conditions at neighboring bottom grid points are coupled, restrictions imposed by hard bottom lying updrift must also be checked. Corrections should be made in the direction of transport.

Significant scour can appear downdrift of an exposed hard-bottom area. For constant waves and water level, it is expected that scour would only continue until some equilibrium depth is attained, after which there would be no further local erosion. However, the model might not properly describe this situation and could overestimate the local scour depending on the hard-bottom configuration. A simple means of limiting local scour downdrift of hard bottom was introduced. It is assumed that the transport rate increases exponentially with distance downdrift of the hard bottom to the potential value, where the spatial rate of increase is determined by an empirical parameter called the scour attenuation coefficient. Based on sensitivity testing, values of the scour attenuation coefficient are expected to lie in the range from 0.1 to 1.0 m^{-1}.

Model Verification

The authors are not aware of field data with which to examine predictions of the model. Data appropriate for comparing with numerical predictions are, however, available in prototype-scale physical model experiments performed in Germany and in smaller "midscale" physical model experiments conducted in the United States. Comparisons were made to these data sets with monochromatic waves.

Large Wave Tank Data

The most suitable data set reported in the literature for testing the hard-bottom calculation algorithm and SBEACH is that of Dette and Uliczka (1986, 1987). They performed experiments on beach profile change in a large wave tank (GWK) in Germany where large waves and realistic beach change can be generated without physical model scaling distortions. The GWK is 324 m long, 7 m deep, and 5 m wide. The sand had a median grain size of 0.33 mm, and the beach was subjected to monochromatic waves with height H = 1.5 m and period T = 6.0 sec. During one case, a significant portion of the sloping cement bottom underlying the sand in the tank was exposed, restricting profile change. Exposure of the cement bottom in this one fortuitous run allows evaluation of the hard-bottom algorithm.

The main calibration parameter is the coefficient K in the sand transport rate equation. The default K value (1.75 10^{-6} m^4/N) produced profile response somewhat slower than the GWK measurements, and K was increased to improve agreement. A value of K = 2.5 10^{-6} m^4/N produced satisfactory agreement. The scour attenuation coefficient was set to 0.2 m^{-1} based on experience gained through numerical experiments. The GWK data provide a severe test for a profile response model because of the steep slope of the initial profile.

Figures 2a and 2b display the initial, calculated, and measured profiles together with the location of the hard bottom (sloping cement false bottom) after 370 and 1,750 waves, respectively. The SBEACH prediction at shorter elapsed time tends to lag movement of material

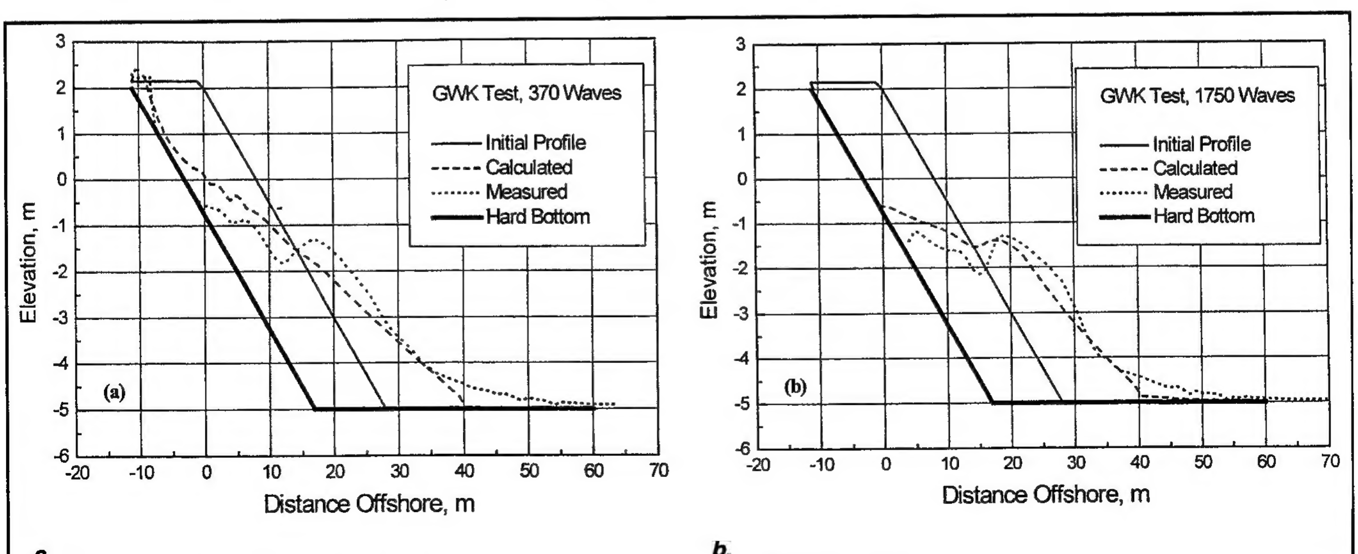

Figure 2. Comparison of SBEACH calculations to profile change in a large physical model

to the offshore, probably because of slumping of the steep slope in the model. The steeply sloping profile prevents development of a bar in the calculations.

After 1,750 waves, the calculated profile has receded enough to expose the concrete bottom. The calculated profile in Figure 2b is close to the equilibrium shape; therefore, a longer simulation time will only cause marginally more erosion and bar buildup. Overall, the final calculated and measured profile (close to equilibrium) supports the applicability of the enhanced SBEACH for accurately predicting how a hard bottom limits the supply of material for transport.

Midscale Wave Tank Data

Comparisons were made to the midscale laboratory experiments of Hughes and Fowler (1990). One of the objectives of their study was to reproduce the GWK results described above in a smaller tank. The physical model was termed "midscale," meaning that the facility and beach were of sufficient size such that the waves generated had heights and periods comparable with the lower end of magnitudes of meaningful field conditions (waves that can produce beach change, such as in the Great Lakes or Gulf of Mexico).

The beach was constructed of a fine-grained sand (0.13-mm median diameter), which tended to reduce scaling distortion. Monochromatic and random waves were run. The monochromatic waves in the midscale model had height H_m = 0.2 m and with period T_m = 2.2 sec. Figures 3a and 3b compare SBEACH calculations and the midscale physical model results. After 370 waves, SBEACH predicts greater recession of the upper profile than the measurements show (note the distinct measured scarp in Figure 3a), and the numerical model did not produce the pronounced offshore bar obtained in the physical model at this elapsed time. The revetment is not yet exposed. However, after 1,850 waves, the entire revetment above still-water level is uncovered, which is well predicted by SBEACH. Also, the model predicts a bar at a location along the profile and with similar dimensions as was measured. The offshore extent of the bar is also well reproduced. An interesting sidelight to the model comparison was that SBEACH replicated the overwash that occurred in some of the physical model tests.

Summary

The SBEACH model was enhanced to account for the presence of a hard bottom. The hard bottom can be specified at any and all locations on the profile. Comparison with a case of hard-bottom exposure measured in a large wave tank and with measurements from several different cases performed in a midscale physical model showed fair to good agreement. Comparisons with the midscale physical-model data also validated the monochromatic- and random-wave transport calculations in SBEACH.

A scaling criterion was derived for the length-scale ratio between prototype and model. Success in reproducing with SBEACH profile change observed in the midscale physical model is an indirect confirmation that the basic physical principles acting to produce storm-induced beach erosion are represented in the numerical model.

SBEACH has been shown capable of calculating storm-induced beach erosion on beaches containing hard bottom. In this capacity, the model is expected to be an aid in design of beach fills and in guiding field-data collection as well as laboratory experiments aimed at investigating the physical processes in the vicinity of hard bottom.

Acknowledgments

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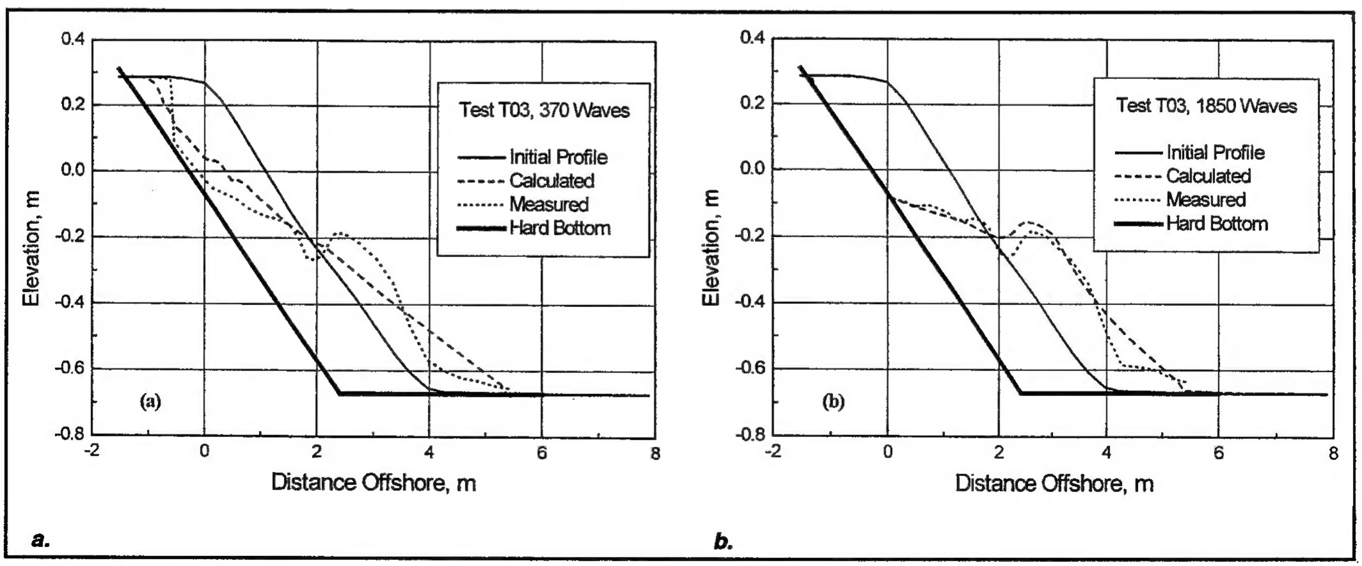

Figure 3. Comparison of SBEACH calculations to profile change in a midscale physical model

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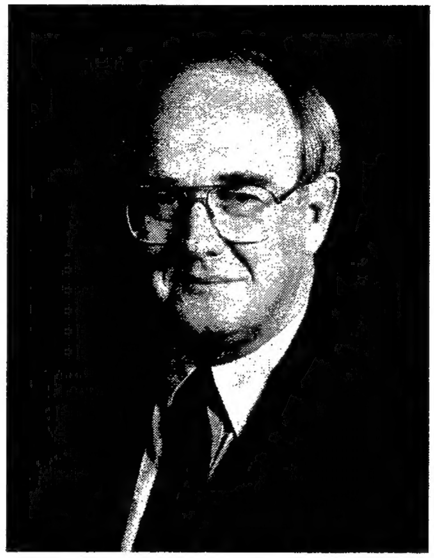
Calendar of Coastal Events

7-10 Jun 1999	**Coastal Structures '99**, Santander, Spain, http://www.omniasc.es/aforo/coastal99, aforo@omniasc.es
20-24 Jun 1999	**Coastal Sediments '99**, Long Island, New York, USA, "Scales of Sediment Motion and Coastal Morphology Change" http://www.coastalsediments.org, n.kraus@cerc.wes.army.mil
24-30 Jul 1999	**Coastal Zone '99**, San Diego, California, USA, "The People; the Coast, the Ocean: Vision 2020" http://omega.cc.umb.edu/~cz99/main.html, cz99@umbsky.cc.umb.edu
8-11 Aug 1999	**1999 International Water Resources Engineering Conference**, Seattle, Washington, USA, seattle1999@westca.com
8-10 Sep 1999	**Breakwaters '99**, Madison, Wisconsin, USA, omagoon@guenoc.com, j prehn@madison.baird.com
23-26 Sep 1999	**California Shore and Beach Preservation Association and Coastal Zone Foundation Conference**, Ventura, California, USA, "Sand Rights '99: Bringing Back the Beaches" omagoon@guenoc.com, kewstone@aol.com, lewing@coastal.ca.gov
14 Oct 1999	**California Coastal Symposium**, Huntington Beach, California, USA, steveaceti@worldnet.att.net
3-5 Nov 1999	**6th International Conference on Estuarine and Coastal Modeling,** New Orleans, Louisiana, USA, spaulding@oce.uri.edu, veritech@magnolia.net
6-9 Mar 2000	**Coast to Coast 2000**, Melbourne, Victoria, Australia vcc@nre.vic.gov.au
1-4 May 2000	**32nd Offshore Technology Conference**, Houston, Texas, USA, rsoussi@spelink.spe.org
16-21 Jun 2000	**27th International Conference on Coastal Engineering**, Sydney, New South Wales, Australia ICEE 2000, P. O. Box N399, Grosvenor Place, Sydney, NSW 2000, Australia

Charles C. Calhoun Retires

Mr. Charles C. Calhoun, Jr., recently retired as Assistant Director of the U.S. Army Engineer Waterways Experiment Station (WES) Coastal and Hydraulics Laboratory (CHL), having completed over 35 years of Federal service all at WES. He began his career in 1963 in the Soils Division (now Geotechnical Laboratory), where he conducted original research on use of geotextile fabrics in drainage facilities. He developed the Tri-Service subsurface drainage manual and became an internationally recognized authority on drainage of pavement systems.

In 1973, Mr. Calhoun moved to the Office of Dredged Material Research (now Environmental Laboratory (EL)) as one of four program managers of the Dredged Material Research Program (DMRP), where he was responsible for engineering aspects. The DMRP was the largest and most diverse research program ever undertaken by the Corps to meet its Civil Works mission and was the cornerstone for continuing research in this area by the Corps. At the conclusion of the DMRP, he became the first manager of the Dredging Operations Technical Support Program. Under his leadership, the Field Verification and Long-Term Effects of Dredging programs were developed and implemented.



As the first manager of the Environmental Effects of Dredging programs, he oversaw all dredging-related research in the EL. In recognition for his contributions to the field of dredging, he was awarded the Moffatt-Nichol Harbor and Coastal Engineering Award by the American Society of Civil Engineers (ASCE).

In 1985, Mr. Calhoun was selected as Assistant Chief of the Coastal Engineering Research Center (CERC), where he led the team that formed the Dredging Research Program. He also oversaw formulation of the Dredging Operations and Environmental Research Program and led the development of the Automated Coastal Engineering System. He steered efforts to implement the Coastal Engineering Education Program. Being responsible for all Coastal Engineering Research Board (CERB) activities in CHL, he formed and participated in numerous CERB task forces including those resulting in the creation of the Coastal Inlets Research Program. Mr. Calhoun continued as Assistant Director of CHL when CERC and the Hydraulics Laboratory merged.

Mr. Calhoun was a facilitator for the Leadership Education and Development program and has trained over 150 Army supervisors. He has chaired two highly successful ASCE technical conferences and is presently planning for a third. In addition to being past chairman of the ASCE Waterways Committee, he is active in the World Dredging Association and has served in leadership positions in the Permanent International Association of Navigation Congresses, one of which being chairman of the Publications Committee for 14 years.

Charlie will be missed by his many personal and professional friends both at WES and around the world. He plans to stay active in the professional community, including conducting leadership development seminars for ASCE.

US Army Corps of Engineers
Waterways Experiment Station

The Corps' Coastal Vision Statement

We will, as the National Coastal Engineer:

- ***Continue our leadership in the protection, optimization, and enhancement of the Nation's coastal zone resources.***
- ***Increase our contribution to the Nation's economy, quality of life, public safety, and environmental stewardship.***

The CERCular

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Contributions of pertinent information are solicited from all sources and will be considered for publication. Communications are welcomed and should be addressed to the U.S. Army Engineer Waterways Experiment Station, Coastal and Hydraulics Laboratory, ATTN: Dr. Lyndell Z. Hales, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199, or call (601) 634-3207, FAX (601) 634-4253, Internet: l.hales@cerc.wes.army.mil

ROBIN R. CABABA
Colonel, Corps of Engineers
Commander

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199

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